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RESULTS OF PARAMETRIC STUDIES PERFORMED FOR THE DEVELOPMENT OF SUPPORT DESIGN GUIDELINES

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OCTOBER 1987

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50 pp

MINING RESEARCH LABORATORIES
DIVISION REPORT MRL 87-128(TR)

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by

M. Gyenge*

ABSTRACT

The dependence of support requirements on rock mass properties was investigated by parametric studies. The stability condition of the various rock masses were assessed by the critical stress and deformation analysis method which is based on rock-support interaction principles.

A procedure for underground support design has been developed. It is based on establishing the allowable radial deformation in relations to the support capabilities of artificial support systems.

The procedures permit assessment of the artificial support potential of specific rock mass conditions. As well, they can be used for the selection of the most economic system. Charts are provided for that purpose. The use of the procedures and charts in the design of an artificial support system are illustrated in a sample design study.

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RÉSULTATS DES ÉTUDES PARAMÉTRIQUES EN VUE DE DÉVELOPPER DES LIGNES DIRECTRICES RELATIVEMENT À LA CONCEPTION DU SOUTÈNEMENT

par

M. Gyenge*

RÉSUMÉ

L'interdépendance des exigences du soutènement et des propriétés des masses rocheuses a fait l'objet d'études paramétriques. L'état de la stabilité des diverses masses rocheuses a été évaluée par la méthode d'analyse de la contrainte critique et de la déformation qui se fonde sur les principes d'interaction du soutènement et de la roche.

Une marche à suivre relativement à la conception du soutènement souterrain a été établie. Elle permet de déterminer la déformation radiale admissible en fonction des capacités de soutènement des systèmes de soutènement artificiels.

La marche à suivre permet d'évaluer le potentiel du soutènement artificiel dans les conditions spécifiques de la masse rocheuse. Elle permet également de sélectionner le système le plus économique et des diagrammes sont présentés dans ce but. L'utilisation des marches à suivre et des diagrammes lors de la conception d'un système de soutènement artificiel est illustrée dans une étude préliminaire de la conception du soutènement.

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INTRODUCTION

In the past two decades many significant developments have taken place in the field of geotechnical engineering. Among the more noticeable is the ability to analyze complex stress distribution and stability problems by computers. The design and evaluation of artificial support systems, based on the concept of rock mass-support system interaction, is another important advancement. Application of this concept is possible because of the present level of ability to analyse and to understand the importance of structural features with respect to the stability of underground excavations.

In spite of such impressive advances, the mining engineer is still faced with major areas of uncertainty when designing an underground support system. One of these uncertainties relates to the failure mechanism that is operative in a jointed rock mass, controlled by its strength properties. Equally, the support capacity of some artificial support systems such as rock bolt and shotcrete need to be better defined. Unlike man-made materials, rock mass conditions are extremely variable and at present can only be poorly characterized. This results in additional uncertainties in support system design and evaluation.

Previous studies have established that existing knowledge and information are sufficient to define the major factors influencing the stability of underground openings in relation to support requirements [1,2]. These studies shed some light on the possible effects of the above uncertainties on the analytical procedures which can take them into consideration.

The relationship between variables however will never be sufficiently precise to permit their expression in the form of mathematical equations. Therefore, the design of a support system for an underground opening will always remain something of an art, based on the application of practical knowledge and experience.

In this report the author has attempted to improve the art

involved in underground support system design by recommending a novel procedure for use in underground support design and evaluation. The procedure is based on the critical stress and deformation analysis method [3], which is an extension of the rock-support interaction analysis method [4].

DEFINITION OF THE PROBLEM

Let us assume that an appropriate support system is to be designed for the horse-shoe shaped drift shown in Figure 1. It is also assumed that the drift has been driven using a full face drill and blast method and that the horizontal and vertical in situ stresses have equal values of p_o.

Support design is a difficult task, because of the large number of interrelated factors involved in rock-support interaction analysis. In order to reduce the difficulties to a manageable level, a number of simplifying assumptions have to be made; these assumptions are:

- Substitution of a two-dimensional circular opening of radius r_i, for the normally horse-shoe shaped drift.
- Equal horizontal and vertical in situ stresses of p_o.
- Uniform radial support pressure requirements of p_i for the walls of opening.
- Linear-elastic material behaviour by the original rock mass which is characterized by the modulus of elasticity E and the Poisson's ratio ν.
- Failure characteristic of an elastic original rock mass defined by the equation:

$$\sigma_1 = \sigma_3 + (m\,\sigma_c\,\sigma_3 + s\,\sigma_c^2)^{1/2} \qquad (1)$$

where, σ_1 = the major principal stress at failure,
σ_3 = the minor principal stress at failure,
σ_c = the uniaxial compressive strength of specimens of the intact rock material,
m and s = material constants which depend upon the rock properties and upon the state of fracturing of the specimen prior to the application of σ_1 and σ_3 stresses.

- Failure characteristic of a perfectly plastic material, for the rock mass following failure, defined by the equation:

$$\sigma_1 = \sigma_3 + (m_r \sigma_c \sigma_3 + s_r \sigma_c^2)^{\frac{1}{2}} \qquad (2)$$

where, m_r and s_r = material constants for broken rock mass.

- Volumetric strains in the elastic region governed by the elastic constants E and ν.

- Volumetric strains in the dilating broken rock governed by the associated flow rule of the theory of plasticity.

- Neither the original nor broken rock mass exhibit time dependent behaviour.

- The radius of the plastic zone depends upon the in situ stress, the support pressure and the material characteristics of both the original and broken rock mass.

The analysis of a specific case requires the following input data:

σ_c = uniaxial compressive strength of intact rock pieces,
E = modulus of elasticity of original rock mass,
ν = Poisson's ratio of original rock mass,
m and s = material constants of original rock mass,

m_r and s_r = material constants of broken rock mass,
y = unit weight of rock mass,
p_o = in situ stress magnitude,
r_i = initial radius of opening.

STABILITY BEHAVIOUR OF ROCK MASS

The stability behavior of a rock mass is assessed by using the load-deformation reaction curves, which have been obtained by the critical stress and deformation analysis method. The principles of analysis and assessment will be explained using Figure 2.

To undertake such an assessment the specific rock mass must be characterized by the input parameters σ_c, m, s, E and ν. For the underground opening defined by the radius of r_i, the various in situ stresses of p_o, p_{o1} and p_{o2} magnitude must be established or estimated. As well, the following material constants of the broken rock mass must be established or estimated: m_{r1}, s_{r1}, m_{r2}, s_{r2} and m_{r3}, and s_{r3}.

With this method of analysis the stability of a particular rock mass is assessed in terms of in situ stress magnitude. If the acting in situ stress is less than the critical in situ stress for the formation, p_{ocr}, (e.g. $p_{o2} < p_{ocr}$ in Figure 2), then the rock mass around the opening will behave elastically. Because no broken zone develops, the opening is stable without support.

The critical in situ stress is calculated using Eq (3).

$$p_{ocr} = \sigma_c \, (s)^{\frac{y}{2}}/2 \qquad (3)$$

The critical deformation ratio u_{icr}/r_i defines the limit of radial elastic deformation of an opening; u_{icr}/r_i is calculated using the following equation:

$$u_{icr}/r_i = \frac{1 + \nu}{E} p_{ocr} \qquad (4)$$

A broken rock zone develops if the acting in situ stress exceeds the magnitude of the critical in situ stress. The development of a broken zone begins at the periphery of the opening, where stress concentrations are greatest. A zone of broken rock will develop only if the support pressure p_i is less than a critical value as determined using equation (5):

$$p_i < p_{icr} = p_o - M\sigma_c \qquad (5)$$

or

$$p_i/p_o < p_{icr}/p_o = 1 - M\sigma_c/p_o \qquad (5a)$$

where,

$$M = \tfrac{1}{2}\left[(m/4)^2 + mp_o/\sigma_c + s\right]^{\frac{1}{2}} - m/8 \qquad (6)$$

Equation (5a) defines the transition curve, which is shown as a heavy line in Figure 2. It begins at the critical radial deformation ratio value of u_{icr}/r_i.

Above the transition curve, where $p_i/p_o > p_{icr}/p_o$, the original rock mass behaves elastically; below this curve, when $p_i/p_o < p_{icr}/p_o$, the broken rock mass behaves plastically.

The transition point of the load-reaction curve for a rock mass under particular load conditions can be easily established. Let us assume, for example, that the in situ stress operative in a specific support design case is p_o. The two coordinates of Point A, the transition point between elastic and plastic regions, are given by equations (5a) and (7):

$$p_i/p_o = p_{icr}/p_o = 1 - M\sigma_c/p_o \qquad (5a)$$

where,

$$M = \tfrac{1}{2}\left[(m/4)^2 + mp_o/\sigma_c + s\right]^{\frac{1}{2}} - m/8 \qquad (6)$$

and

$$u_{icr}/r_i = \frac{1 + \nu}{E} M\sigma_c \qquad (7)$$

In the elastic region, above Point A, the load-deformation reaction curve is linear. Below Point A the load-deformation reaction curve is a curved line, because of the plastic behaviour of the broken rock zone. The plastic deformation of the broken rock zone, as demonstrated in Figure 2, is governed by the material constants m_r and s_r.

The points of the curved line portion of a load-deformation curve is calculated for any value of p_i, within the required range of $p_{icr} > p_i > 0$, by the following set of equations:

$$D = \frac{-m}{m + 4\,[m/\sigma_c\,(p_o - M\sigma_c) + s]^{1/2}} \tag{8}$$

$$N = 2\left[\frac{p_o - M\sigma_c}{m_r\sigma_c} + \frac{s_r}{m_r^2}\right]^{1/2} \tag{9}$$

$$\frac{r_e}{r_i} = e^{\,N - 2\left(\frac{p_i}{m_r\sigma_c} + \frac{s_r}{m_r^2}\right)^{1/2}} \tag{10}$$

$$\text{for } r_e/r_i < (3)^{1/2} \quad R = 2\,D\,\ln\, r_e/r_i \tag{11}$$

$$\text{for } r_e/r_i < (3)^{1/2} \quad R = 1.1\,D \tag{12}$$

$$e_{av} = \frac{2\,[u_e/r_e]\,[r_e/r_i]^2}{[\,[r_e/r_i]^2-1]\,[1+1/R]} \tag{13}$$

$$A = (\,2\,u_e/r_e - e_{av})\,(r_e/r_i)^2 \tag{14}$$

$$\frac{u_i}{r_i} = 1 - \left[\frac{1 - e_{av}}{1 + A}\right]^{1/2} \tag{15}$$

All terms in the above equations have been previously defined with exception of the following two:

u_e = radial displacement of the elastic boundary,

r_e = radius of the broken zone.

SUPPORT REQUIREMENTS

The fundamental principle that must be understood in designing an underground support system is that the rock itself must provide the major structural element. Artificial support elements are used to help the rock mass to support itself. In some cases the stability of underground opening is achieved entirely through use of the self-supporting capability of the rock material. In other cases, additional structural materials such as steel, concrete, wood, etc. are used to provide reinforcement.

Support system have to meet a number of criteria. The most important criterion is adequate safety. Other criteria are related to economic, functional and operational considerations.

On the basis of such criteria the best system is selected to reinforce the unstable or potentially unstable rock around an opening. The aim is to achieve a balanced design, where, the strength capacities of all structural elements, including the rock, are fully utilized to achieve stability with a reasonable safety margin.

The effectiveness of a support system can be assessed through observing the deformation of the opening, as it revealed by fracturing or non-fracturing of surface rocks. Some relatively minor amount of spalling and fracturing is tolerated in most mining applications without the need for additional support. Support will be required for stability if the radial deformation of the unsupported opening is larger than u_{ult}, which is defined as:

u_{ult} = allowable radial deformation causing only tolerable spalling and fracturing problems.

The selection of a support system for a particular site can be based on use of the applicable pressure-deformation curve. The utilization is demonstrated in Figure 3, with three different conditions. The support pressure-deformation curves are representative of the types of load-deformation reaction curves of Figure 2. They are applicable for the specific opening size defined by r_i and the operative in situ stress conditions defined by p_o.

In Figure 3(a) the underground opening is subjected to an in situ stress p_o which is smaller than the critical in situ stress of p_{ocr}. Consequently, the entire rock mass around of the opening is in elastic state. The opening is stable without support; no broken rock zone develops. The radial deformation is much less than the allowable radial deformation of u_{ult}.

Figure 3(b) represents the case where the in situ stress is larger than the critical in situ stress (i.e. $p_o > p_{ocr}$). Therefore, a broken rock zone will occur. The support pressure-radial deformation curve consists two sections. The linear section above the transition point A correponds to elastic deformation of the rock mass. The ratio of p_i/p_o for this section is larger than p_{icr}/p_o.

The non-linear section of the support pressure-radial deformation curve is resulted by the plastic behaviour of a broken rock mass. The ratio p_i/p_o for this non-linear section is smaller than p_{icr}/p_o. A stable condition for an opening will be reached at a radial deformation value of u_o with $p_i = 0$. The region where $u_o > u_{ult}$ signifies unstable underground condition producing unacceptable levels of fracturing and spalling problems.

An appropriate and properly installed support system with a supporting capacity of p_s will, however, stabilize the underground opening at a radial deformation of u_s.

The effect of further increased in situ stress, p_o, is demonstrated in Figure 3(c). The support pressure-radial deformation curve now has higher values of p_i and u_i than before. The previously used support system of p_s capacity will not be effective any more, because $u_s > u_{ult}$. The support system, however, could stabilize the underground opening, but only at a radial deformation which would be untolerable in terms of fracturing the opening. A support system with a higher capacity and stiffness would be required.

SELECTION OF SUPPORT SYSTEM

As illustrated previously the load-deformation reaction curve can be used in the selection of a support system. So far the discussion has concerned with the establishment of support system capacity. There are, however, other factors that must be taken into consideration in the selection of a support system. Figure 4 is used to outline these additional factors.

With respect to support system selection, the support pressure-radial deformation curve in Figure 4 is divided into three sections. The first section which is the elastic region starts with the coordinates $u_i = 0$ and $p_i = p_o$ (these coordinates are not shown in Figure 4) and terminates at point A, with coordinates of $u_i = u_e$ and $p_i = p_{icr}$.

The second section is between points A and P of the non-linear portion of the support pressure-radial deformation curve, with acceptable levels of radial deformation in terms of the use of support systems. The coordinates of point P are $u_i = u_{ult}$ and $p_i = p_{ib}$.

The third section is the non-linear portion of the p_i vs u_i curve beyond point P, where radial deformation exceeds the allowable radial deformation limit of u_{ult}.

The first section describes elastic stress-strain conditions, for which support is not required. Support selection for the third section

need not be considered because the radial deformation exceeds u_{ult}. Support systems need only be considered for the conditions defined by the second section of the p_i vs u_i curve, i.e. between points A and P.

In Figure 4, for purpose of demonstration, three support systems are contemplated: L, M and N. System L, with a support capacity of p_{il}, will stabilize the failing rock mass with the occurence of a radial deformation of u_{il}, provided that it was installed when the radial deformation was only $\bar{u}_{il}$. The slope between points $\bar{u}_{il}$ and 1 represents the load-deformation characteristic of support system L. L is the stiffest of the three systems, as indicated by having the steepest slope.

Support system M has a lower support capacity, equal to p_{im}. This system will stabilize the failing rock mass at a radial deformation value of u_{im}, provided that it was installed when the radial deformation was $\bar{u}_{im}$. It represents a support system with an intermediate stiffness.

Support system N has the lowest support capacity of the three systems: p_{in}. It is the least stiff system of the three, as indicated by the lowest load-deformation slope. The performance of support system N would be unsatisfactory, because its stabilizing effect would only occur at a radial deformation value of u_{in} which exceeds u_{ult}. A support system can be effectively applied only if $u_s < u_{ult}$.

On the basis of support capacity and radial deformation requirements systems L and M are equally suitable. In practice the final decision would be based on relative cost.

As a general rule, support systems with the lower capacity are less expensive. Consequently, system M is the probable economic choice. A support system is economically feasible if $u_e \geq u_s \leq u_{ult}$. A support system is not economical if $u_s < u_e$, because, in this case the installed system would impede non-elastic deformations of the rock. Any attempt to prevent non-elastic deformation completely, would be totally uneconomic.

Installation techniques must also be considered in selecting a support system. Some types of support can be installed right at the face, while others can only be installed several meters behind the face. With reference to Figure 4, since $\bar{u}_{im} > \bar{u}_{il}$, the acceptable installation distance from the face is larger for support system M than for support system L. Usually, the greater the installation distance from the face the easier the installation procedures become. For this reason, support system M would again be the probable choice.

PROCEDURE OF SUPPORT SELECTION

SIMPLIFIED LOAD-DEFORMATION RELATIONSHIP

The coordinates for points within the non-linear portion of the load-deformation reaction curve can be calculated using equations (8) through (15). The required calculations are, however, rather lengthy.

Investigations of various support design cases, involving an entire range of rock types and in situ stresses, resulted in the development of a simplified procedure summarized in Figure 5.

The simplification is based on substituting a straight line for the non-linear portion of the load-deformation reaction curve. For the substitution to be effectively used, however, a scaled plot is essential. In Figure 5(a) the percentage values of the normalized radial deformation (u_i/r_i) are plotted along the abscissa and the normalized values of support pressure p_i are plotted along the ordinate in the same scale, i.e., $0.1\ (u_i/r_i)\% = 0.1\ (p_i/p_o)$.

With the scale plot the non-linear regions of the load-deformation reaction curve can be treated as the long side of a right-angled triangle, as shown in Figure 5(a) by line AB. The 90° vertex of the triangle is at the transition point A. The other side of the triangle is between point A and the origin of the transition curve, which is marked in Figure 5(a) by point C. The hypotenuse of the triangle is defined by the distance between points C and B, along the abscissa.

As shown in Figure 5(a), the straight line substitution increases the radial deformation. This increase has the same order of magnitude as the time-dependent radial deformation components, established by field measurements in tunnels [8,9]. The total radial deformation is the sum of the instantaneous (calculated non-linear load- deformation curve) and the time-dependent components.

From the geometrical relationships in Figure 5(a), it follows that:

$$a = (u_i/r_i - u_{icr}/r_i)100 \tag{16}$$

thus the acute angle at vertex C is:

$$\alpha = \arctan \frac{p_{icr}/p_o}{a} \tag{17}$$

The other acute angle at vertex B is:

$$\beta = 90 - \alpha \tag{18}$$

and thus

$$b = \frac{p_{icr}/p_o}{\tan \beta} /100 \tag{19}$$

The abscissa of vertex B is calculated by the following equation:

$$\bar{u}_o/r_i = u_i/r_i + b \tag{20}$$

The critical support pressure, corresponding to an in situ stress magnitude of p_o, is:

$$p_i = p_{icr} = (p_{icr}/p_o)\, p_o \tag{21}$$

Further calculation steps are demonstrated by referring to the right-angled triangle in Figure 5(b), which has a base of b and a height of p_{icr}.

Let p_s be the support capacity of the support system under consideration. From the proportion of the triangles in Figure 5(b) it follows that:

$$c = \frac{p_s\, b}{p_{icr}} \tag{22}$$

Then the supported radial deformation, in terms of real deformation, is:

$$u_s = (\bar{u}_o/r_i - c)\, r_i \qquad (23)$$

The support systems under consideration with a support capacity of p_s can effectively be used if $u_s \leq u_{ult}$.

SUPPORTED RADIAL DEFORMATION - DEPTH (IN SITU STRESS) CURVES

From the previous discussions it is evident that support pressure-radial deformation curve, and consequently the supported radial deformation, u_s, varies with in situ stress, p_o.

The support pressure-radial deformation curve also varies with rock type, rock strength and deformation properties, rock mass quality, and diameter of the underground opening.

The relationship between these interacting variables can conveniently be expressed by the supported radial deformation versus in situ stress curves (u_s vs p_o). Alternatively, because of the assumption of equal vertical and horizontal stresses, the relationships can be represented by deformation versus depth curves (u_s vs d). The in situ stress gradient in the latter case is the average unit weight of the rock mass (kg/m^3), expressed in the terms of MPa/m.

In Figure 6, for example, the u_s vs d (or u_s vs p_o) curves for rock bolt, shotcrete, concrete and steel sets supports are presented. The curves in Figure 6 are valid for use in a specific design case, where: the rock masses are of very good quality (m = 3.5 and s = 0.1), the strength and deformation properties are: σ_c = 100 MPa, E = 37 GPa and ν = 0.25, and the radius of the underground opening is r_i = 2.5 m. The numbers in Figure 6 refer to the Designation numbers in Table 1.

A rock mass having the above characteristics can be considered as self-supporting for all opening sizes down to a depth of 586 m. This depth is equivalent to a critical in situ stress of p_{ocr} = 15.81 MPa, if the assumed average unit rock weight is 2.75 x 10^3 kg/m^3. Below this depth openings will need to be supported.

The starting point for the u_s vs d curves is at the depth of 586 m. The limiting depth for an effective support is given by the depth where the radial deformation value of the u_s vs d curve is equal to u_{ult}.

Let us assume that the allowable radial deformation for this rock mass is 20 mm. The vertical line, which represents u_{ult} = 20 mm, intersects the u_s vs d curve for rock bolt support at a depth of 1200 m (Figure 6(a)). This intersection point defines the limiting depth for the effective use of rock bolt support for an underground opening excavated with a 2.5 m radius, in this particular rock mass.

The curves in Figure 6(b) refer to two types of shotcrete. The line of u_{ult} = 20 mm intersects these curves at 1160 m and 1330 m depths. Beyond these limiting depths shotcrete lining would not provide effective support, using these mixes.

For the heavier support systems the calculations could result negative supported radial deformation values; such as shown in Figure 6(c). It is an indication of excess supporting capacity. The particular system with negative values of u_s, under the given design conditions, would provide an over-designed and therefore uneconomic support. The same support may, however, become economic at depths below d_{min}, which is defined by the intersection point of u_s vs d curve with the vertical axis of u_s = 0.

DESIGN CHARTS

The information contained in radial deformation-depth curves can also be provided in the form of design chart. Appropriately structured design charts are probably the simplest, and at the same time, the most

effective means to provide guidance in support selection cases using the presented analytic method.

Figure 7 is the design chart presentation of the information in Figure 6. Use of the chart is simple, as corroborated by the following discussions.

The given rock mass is self-supporting down to a depth of 586 m (equivalent to 15.81 MPa in situ stress). The chart indicates that rock bolts, can be used as effective support to a limiting depth of 1200 m (33 MPa). If a shotcrete lining is used the limiting depth is either 1160 m (31.9 MPa) or 1330 m (36.8 MPa), depending upon the shotcrete mix used. While technically possible, the use of a 30 cm thick concrete lining for depth less than 1080 m (29.7 MPa) would be uneconomic. However, it could provide effective and economic support between depths of 1080 m and 1860 m (51.2 MPa).

Below its limiting depth a particular support system cannot be used. Either another support system, which is effective, must be selected or mining methods with different support requirements must be used.

TYPICAL DESIGN DATA

A specific support design problem should normally be addressed and support design undertaken with site specific design data available. For the purpose of a design guidance, however, typical average data values can be used.

ROCK PROPERTIES

The rock properties vary with the rock type. According to Hoek and Brown for support considerations rocks can be partitioned into five groups [4]. The rock types within a group are assumed to have similar mechanical behaviour.

The five groups are:

Type A - the carbonate rocks with well developed crystal cleavage, such as: dolomite, limestone and marble;

Type B - the lithified argillaceous rocks, such as: mudstone, siltstone and shale;

Type C - the arenaceous rocks with strong crystals and poorly developed crystal cleavage, such as: sandstone and quartzite;

Type D - the fine grained polyminerallic igneous crystalline rocks, such as: andesite, dolerite, diabase and rhyolite; and

Type E - the coarse grained polyminerallic igneous and metamorphic crystalline rocks, such as: amphibolite, gabbro, gneiss, granite, norite and quartzdiorite.

For the purpose of this study it was assumed that the elastic modulus, E, is linearly related to compressive strength σ_c. The paired values of σ_c and E, based on this relationship, are listed in Table 1.

Table 1 - List of paired uniaxial compressive strength and elastic modulus values

Rock Type									
A		B		C		D		E	
σ_c, MPa	E, GPa	σ_c, MPa	E, GPa	σ_c, MPa	E, GPa	σ_c, MPa	E, GPa	σ_c, MPa	E, GPa
50	27	50	27	50	27	100	37	100	37
100	37	100	37	100	37	200	58	200	58
200	58	200	58	200	58	300	78	300	78
		300	78	300	78	400	99	400	99
				400	99			450	109

The value of the Poisson's ratio for rock varies between 0.15 and 0.30. Sensitivity analysis with respect to Poisson's ratio has revealed that Poisson's ratio has only limited effect on analytic results. Consequently, it is reasonable to use a constant value of $\nu = 0.25$.

The rock mass qualities, for each rock types, are defined by Hoek and Brown as follows [4]:

VERY GOOD QUALITY ROCK MASS

Tightly interlocking undisturbed rock with unweathered joints spaced at ± 3 m.

GOOD QUALITY ROCK MASS

Fresh to slightly weathered rock, slightly disturbed with joints spaced at 1-3 m.

FAIR QUALITY ROCK MASS

Several sets of moderately weathered joints spaced at 0.3-1 m.

POOR QUALITY ROCK MASS

Numerous weathered joints spaced at 30-500 mm with some gouge filling/clean waste rock.

VERY POOR QUALITY ROCK MASS

Numerous heavily weathered joints spaced less than 50 mm with gouge filling/waste rock with fines.

The material constant values, related to the rock types and rock mass qualities, are listed in Table 2.

Table 2 - List of material constants

Rock Type	Rock Mass Quality									
	Very Good		Good		Fair		Poor		Very Poor	
	m	s	m	s	m	s	m	s	m	s
A	3.5	0.1	0.7	0.004	0.14	0.0001	0.04	0.00001	0.007	0
B	5.0	0.1	1.0	0.004	0.20	0.0001	0.05	0.00001	0.010	0
C	7.5	0.1	1.5	0.004	0.30	0.0001	0.08	0.00001	0.015	0
D	8.5	0.1	1.7	0.004	0.34	0.0001	0.09	0.00001	0.017	0
E	12.5	0.1	2.5	0.004	0.50	0.0001	0.13	0.00001	0.025	0

The radii of the openings were selected to correspond to the commonly used mining drift and stope sizes. Therefore, the following radii were considered: 1 m, 2.5 m, 5.0 m and 10.0 m.

CAPACITY OF SUPPORT SYSTEMS

The capacities of the various support systems, other than rock bolts, are summarized in Table 3. The listed values are based on the maximum support pressures, as given by Hoek and Brown [4].

Table 3 - Support system capacities

SUPPORT SYSTEM		Capacity, p_s in MPa			
SPECIFICATION	DESIG-NATION NO.	r_i = 1.0 m	r_i = 2.5 m	r_i = 5.0 m	r_i = 10.0 m
SHOTCRETE - 5 cm thick, $\sigma_{c\ conc}$ = 14 MPa after 1 day	1	0.65	0.27	0.14	0.07
SHOTCRETE - 5 cm thick $\sigma_{c\ conc}$ = 35 MPa after 28 days	2	1.63	0.68	0.34	0.17
CONCRETE - 30 cm thick, $\sigma_{c\ conc}$ = 35 MPa after 28 days	3	7.14	3.55	1.93	1.00
CONCRETE - 50 cm thick, $\sigma_{c\ conc}$ = 35 MPa after 28 days	4	9.72	5.35	3.04	1.63
STEEL SETS - 6I12, 2 m spacing, blocked 2 θ = 22.5° σ_{ys} = 248 MPa	5	0.61	0.18	0.07	0.02
STEEL SETS - 8I23, 1.5 m spacing, blocked 2 θ = 22.5° σ_{ys} = 248 MPa	6	1.59	0.50	0.18	0.06
STEEL SETS - 12W65, 1.0 m spacing, blocked 2 θ = 22.5° σ_{ys} = 248 MPa	7	7.28	2.53	1.04	0.38

The support capacity of rock bolts is based on the pull test results, obtained in various hard rock mines in Ontario [5,6]. The support capacities of the 16 mm (5/8 in) diameter mechanically anchored rock bolts, for various bolting patterns, are summarized in Table 4.

Table 4 - Support capacity of 16 mm (5/8 in) rock bolts

Bolting pattern	p_s, in MPa
1.0 m x 1.0 m	0.055
1.5 m x 1.5 m	0.024
2.0 m x 2.0 m	0.014
2.5 m x 2.5 m	0.009

The support capacity of bolt systems, unlike the capacity of other systems, is independent of the radius of the opening. Furthermore with the very low support pressures involved, the effectiveness of a rock bolt support must be attributed to other mechanisms than the support pressure of p_s provided by the bolts. The previously described procedure of support selection, however, is applicable for the rock bolt. The only difference is that in this case the supported radial deformation u_s (Figure 5) is equal to $\bar{u}_o$. It does not matter what the bolting pattern is.

ALLOWABLE RADIAL DEFORMATION

From the foregoing discussions it is apparent that the recommended procedure for underground support design and evaluation, while using established design principles, represents a novel approach in several

concepts. The critical stress and deformation analysis method and the concept of the supported radial deformation-depth curve represent two new design concepts.

The introduction of the allowable radial deformation concept as the criterion to estimate system effectiveness represents the third and perhaps the most significant novelty within the presented support design and evaluation procedures.

Deformation measurement represents the most advanced, reliable and frequent measurement technique used in rock mechanics field investigations. Meaningful and reliable evaluation of support performance should, therefore, involve deformation measurement of underground opening.

The most logical criterion to evaluate the effectiveness of an underground support design is its performance, in terms of allowable radial deformation, defined as:

u_{ult} = allowable radial deformation causing only tolerable spalling and fracturing problems.

Although deformation of underground openings and the surrounding rocks are frequently measured, published data on the relationship between the stability conditions of supported and un-supported openings and measured deformations are rare. Therefore, until sufficient data are generated, estimated allowable radial deformation values must be used.

A value of u_{ult} = 20 mm, was chosen, albeit somewhat conservative, from measurements reported by Bieniawski [7], after Cording. The tolerable displacement, observed in large underground chambers, were in the order of 12 to 75 mm. The range of displacement in the case of the Washington Metro was in the order of 20 to 50 mm.

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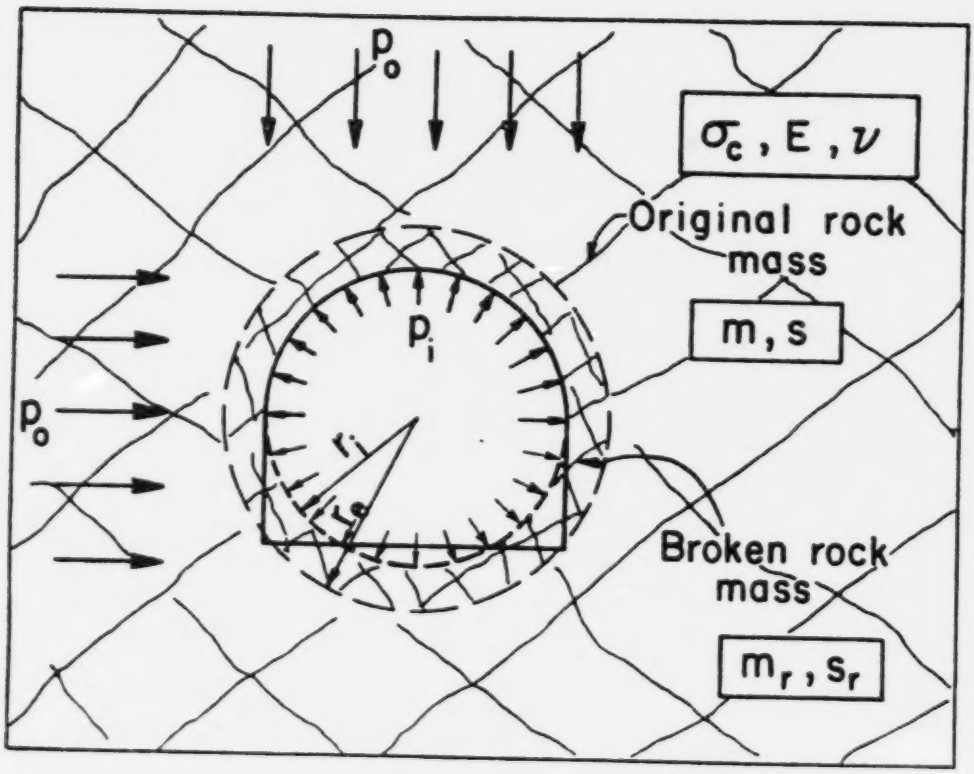


Fig. 1 - Simplified cross-section of the analysed underground opening.

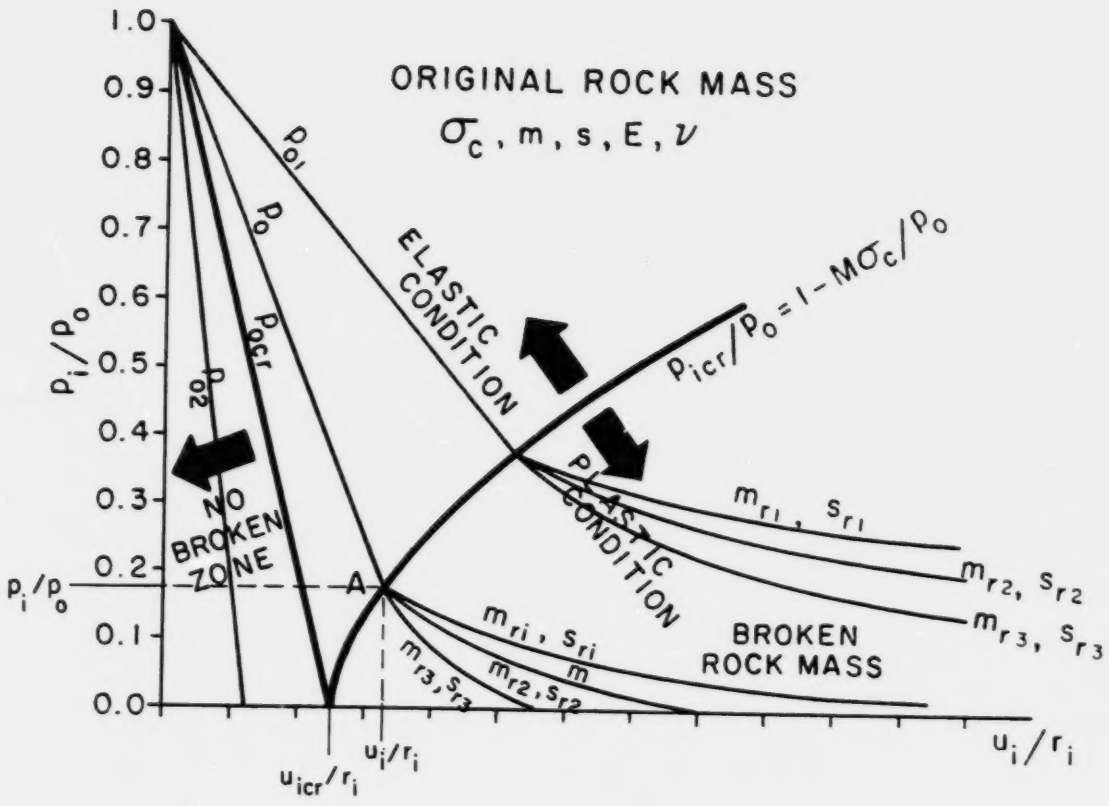


Fig. 2 - The expected stability behaviour of a rock mass based on critical stress and deformation analysis.

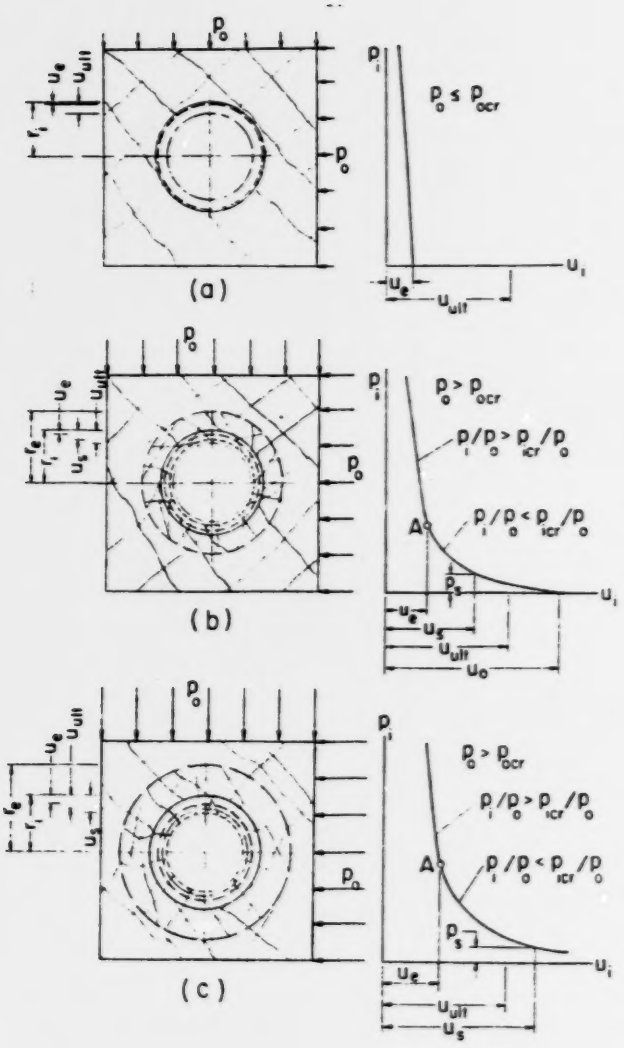


Fig. 3 - Relationship between the radial deformation and support requirement.

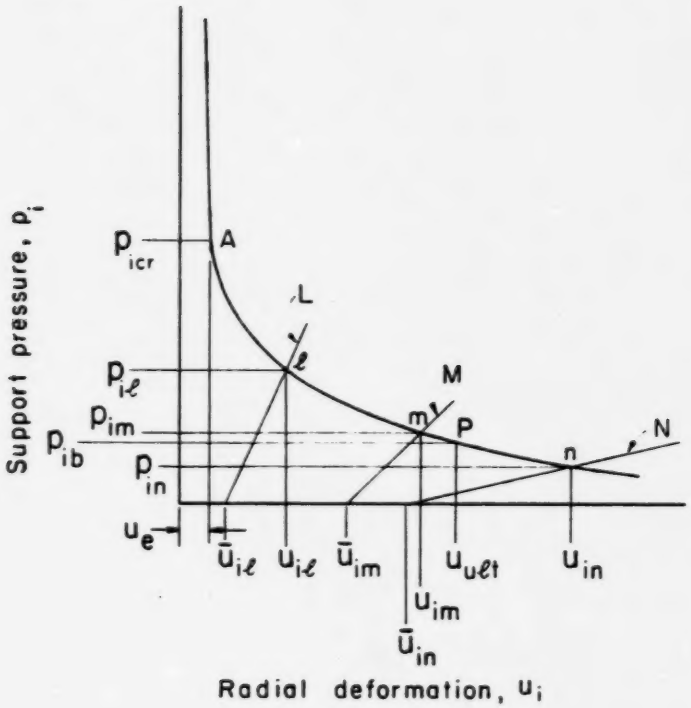
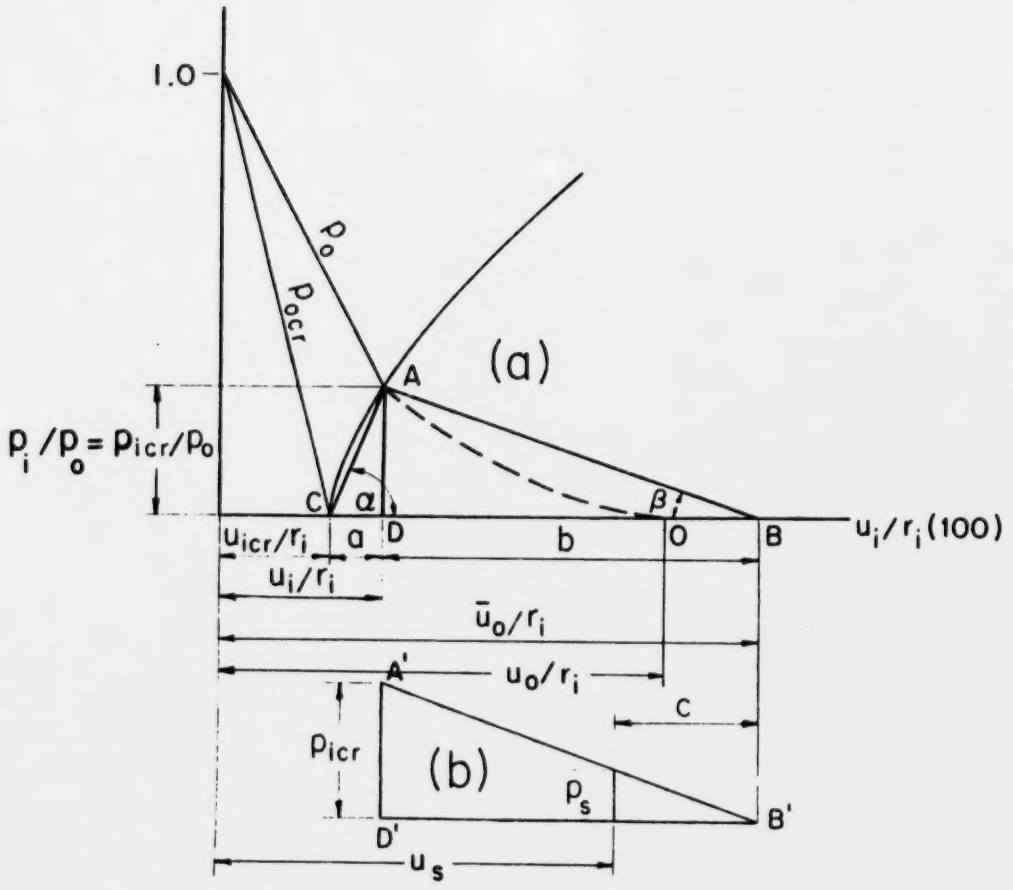


Fig. 4 - Support selection principles.



$$a = (u_i/r_i - u_{icr}/r_i)\,100 \quad (16)$$

$$\alpha = \tan^{-1}\frac{p_{icr}/p_o}{a} \quad (17)$$

$$\beta = 90 - \alpha \quad (18)$$

$$b = \left(\frac{p_{icr}/p_o}{\tan\beta}\right)/100 \quad (19)$$

$$\bar{u}_o/r_i = u_i/r_i + b \quad (20)$$

$$p_{icr} = (p_i/p_o)\,p_o \quad (21)$$

$$c = \frac{p_s\, b}{p_{icr}} \quad (22)$$

$$u_s = (\bar{u}_o/r_i - c)\,r_i \quad (23)$$

Fig. 5 - Procedure of support system evaluation.

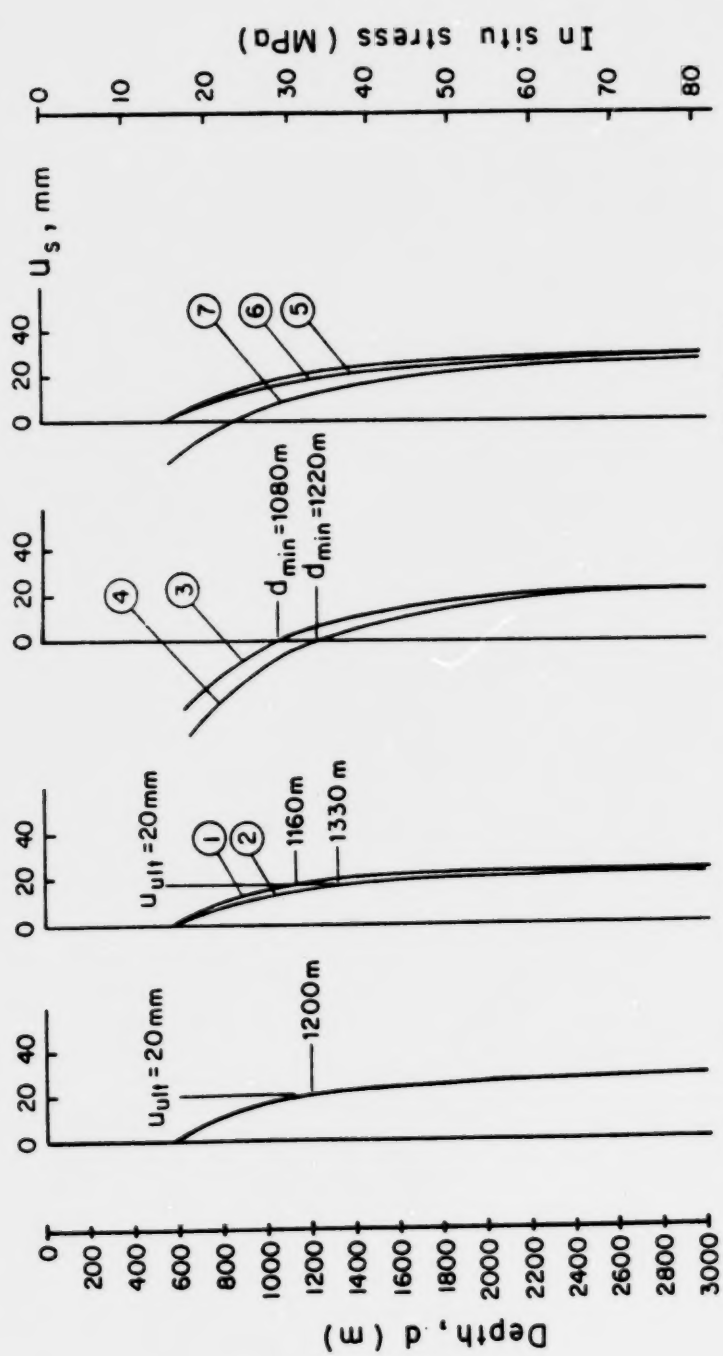


Fig. 6 - Supported radial deformation - depth (in situ stress) curves.

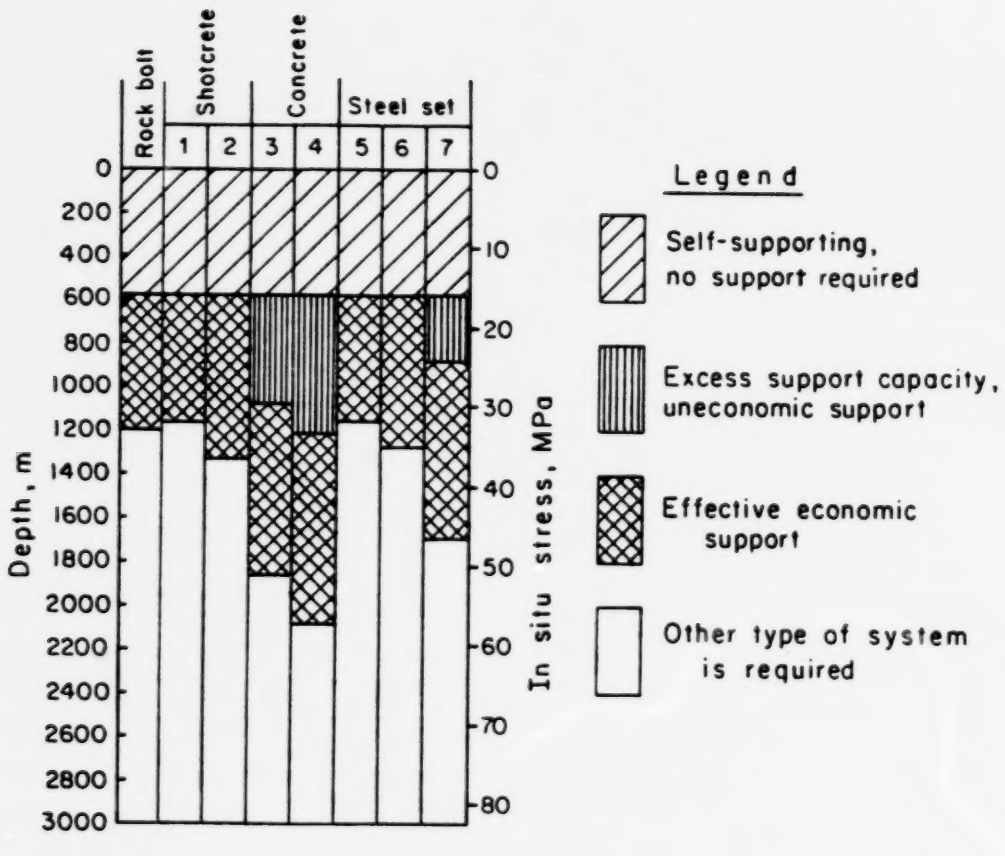


Fig. 7 - Support evaluation and design chart.

APPENDIX

In order to demonstrate the details of the described procedure of underground support design and evaluation, an example had been worked out. It provides the steps of detailed calculation, as well as, graphical presentation.

Let us assume that the mining operation is within a limestone formation. For the purpose of support selection the stability condition of the rock mass around the opening is to be determined as function of vertical depth. The analyses to be performed for the underground openings with 1.0, 2.5, 5.0 and 10.0 m radii. Then, the effective and economic depths for the traditional support systems are to be established; and summarized in design chart form which can be used as a guide for support system selection.

The rock mass quality of the formation is very good. The assumed quality of the broken rock mass is good. The uniaxial compressive strength of the rock is 100 MPa and the Poisson's ratio is 0.25. The unit weight of the rock mass is 2.75×10^3 kg/m^3.

The summarized input data are:

Rock Type: A

σ_c = 100 MPa

E = 37 GPa (from Table 1)

ν = 0.25

m = 3.5

s = 0.1

m_r = 0.7 (from Table 2)

s_r = 0.004

γ_a = 2.75×10^3 kg/m^3

r_i = 1.0, 2.5, 5.0, 10.0 m

p_o = increases at a gradient of 0.0275 MPa/m

The critical in situ stress is calculated by Eq(3):

$$p_{ocr} = \sigma_c \, (s)^{1/2}/2 \qquad (3)$$

$$p_{ocr} = 100 \, (0.1)^{1/2}/2 = 15.81 \text{ MPa}$$

and then the critical radial deformation ratio is by Eq (4):

$$u_{icr}/r_i = \frac{1 + \nu}{E} \, p_{ocr} \qquad (4)$$

$$u_{icr}/r_i = \frac{1 + 0.25}{E} \, 15.81 = 5.34 \times 10^{-4}$$

The origin of the transition curve, as shown in Figure A-1(a), is defined by this value of critical radial deformation ratio. The straight line between this and the $p_i/p_o = 1.0$ points represents the elastic ray for the critical in situ stress of $p_{ocr} = 15.81$.

The coordinates of any desired point of the transition curve are calculated by using Eqs (6), (5a) and (7). As an example, these coordinates were calculated for the in situ stress value of $p_o = 50$ MPa.

$$M = 1/2 \left[\left(m/4 \right)^2 + m p_o/\sigma_c + s \right]^{1/2} - m/8 \qquad (6)$$

$$M = 1/2 \left[\left(3.5/4 \right)^2 + 3.5 \times 50/100 + 0.1 \right]^{1/2} - 3.5/8 = 0.3711$$

$$p_i/p_o = p_{icr}/p_o = 1 - M\sigma_c/p_o \qquad (5a)$$

$$p_i/p_o = 1 - 0.3711 \times 100/50 = 0.2577$$

$$u_i/r_i = \frac{1 + \nu}{E} \, M\sigma_c \qquad (7)$$

$$u_i/r_i = \frac{1 + 0.25}{37,000} \left[0.3711 \times 100 \right] = 12.54 \times 10^{-4}$$

Similar calculations were repeated for in situ stress values of 40 MPa and 30 MPa. The results of calculations are plotted in Figure A-1(a).

For further analysis the scales of plotting are essential. The normalized radial deformation values (u_i/r_i) to be plotted on the abscissa in percentage scale. The scale of the ordinate axis is to be set in such manner that the lengths of the equal numerical values are the same, i.e., 0.1 $(u_i/r_i)\%$ = 0.1 (p_i/p_o).

The non-linear portion of the load-deformation reaction curve begins at the interesection point of the elastic ray of the in situ stress in question and the transition curve. In Figure A-1(b) the intersection point, corresponding to the in situ stress of p_o = 50 MPa, is marked by point A.

Although, the non-linear portion of the load-deformation reaction curve is not required in the recommended support selection procedure, here, for demonstration purposes, it is shown in Figure A-1(b) by solid line. It is the plot of the calculated normalized radial deformation values (u_i/r_i) obtained for various arbitrary stress ratio values (p_i/p_o). The required and rather lengthy calculation steps are demonstrated for the stress ratio value of p_i/p_o = 0.1.

$$D = \frac{-m}{m + 4 \left[m/\sigma_c \left(p_o - M\sigma_c \right) + s \right]^{1/2}} \qquad (8)$$

$$D = \frac{-m}{3.5 + 4 \left[3.5/100 \left(50 - 0.3711 \times 100 \right) + 0.1 \right]^{1/2}} = -0.541$$

$$N = 2 \left[\frac{p_o - M\sigma_c}{m_r \sigma_c} + \frac{s_r}{m_r^2} \right]^{1/2} \qquad (9)$$

$$N = 2\left[\frac{50 - 0.3711 \times 100}{0.7 \times 100} + \frac{0.004}{0.07^2}\right]^{1/2} = 0.877$$

$p_i/p_o = 0.1$, then $p_i = 0.1 \times 50 = 5$

$$\frac{r_e}{r_i} = e^{N - 2\left(\frac{p_i}{m_r \sigma_c} + \frac{s_r}{m_r^2}\right)^{1/2}} \qquad (10)$$

$$\frac{r_e}{r_i} = e^{0.877 - 2\left(\frac{5}{0.7 \times 100} + \frac{0.004}{0.7^2}\right)^{1/2}} = 1.367$$

because $r_e/r_i = 1.367 < (3)^{1/2}$

$$R = 2\,D \ln r_e/r_i \qquad (11)$$

$$R = 2\,(-0.541)\ln 1.367 = -0.338$$

$$e_{av} = \frac{2\,(u_e/r_e)\,(r_e/r_i)^2}{\left[(r_e/r_i)^2 - 1\right](1 + 1/R)} \qquad (13)$$

$$e_{av} = \frac{2\,(12.54 \times 10^{-4})\,(1.367)^2}{\left[(1.367)^2 - 1\right](1 - 1/0.338)} = -27.54 \times 10^{-4}$$

$$A = (2\,u_e/r_e - e_{av})\,(r_e/r_i)^2 \qquad (14)$$

$$A = (2 \times 12.54 \times 10^{-4} + 27.54 \times 10^{-4})\,(1.367)^2 = 98.33 \times 10^{-4}$$

$$\frac{u_i}{r_i} = 1 - \left[\frac{1 - e_{av}}{1 + A}\right]^{1/2} \qquad (15)$$

$$\frac{u_i}{r_i} = 1 - \left[\frac{1 + 27.54 \times 10^{-4}}{1 + 98.33 \times 10^{-4}}\right]^{1/2} = 35.11 \times 10^{-4}$$

For the purpose of support selection, the non-linear portion of load-deformation reaction curve (A0) is substituted by a straight line (AB), as shown in Figure A-1(b) by broken line. In the scale of Figure A-1(b), line AB is the longer side of a right-angled triangle. The 90° vertex of the triangle is at point A. The shorter side is between points A and D, while the hypotenuse is defined by the distance between points C and B.

The necessary steps of calculation are as follows (see Figure 5 for nomenclature):

$$a = (u_i/r_i - u_{icr}/r_i)\ 100 \tag{16}$$

$$a = (12.54 \times 10^{-4} - 5.34 \times 10^{-4})\ 100 = 0.072$$

$$\alpha = \arctan \frac{p_{icr}/p_o}{a} \tag{17}$$

$$\alpha = \arctan \frac{0.2577}{0.072} = 74.39°$$

$$\beta = 90° - \alpha \tag{18}$$

$$\beta = 90° - 74.39° = 15.61°$$

$$b = \frac{p_{icr}/p_o}{\tan \beta} / 100 \tag{19}$$

$$b = \frac{0.2577}{\tan 15.61} / 100 = 92.24 \times 10^{-4}$$

$$\bar{u}_o/r_i = u_i/r_i + b \tag{20}$$

$$\bar{u}_o/r_i = 12.54 \times 10^{-4} + 92.24 \times 10^{-4} = 104.78 \times 10^{-4}$$

The further calculation steps are based on support pressure-radial deformation curve which is obtained from the load-deformation reaction curve for the specific value of in situ stress p_o and for the specific opening size of r_i. The in situ stress, in the example used for the demonstration, is p_o = 50 MPa. Let us also assume that the radius of the opening in question is r_i = 2.5 m.

The support pressure-radial deformation curve for these specific values is shown in Figure A-1(c). Line A'B' is equivalent to the simplified load-deformation reaction curve of line AB of Figure A-1(b). The calculated support pressure and radial deformation values for the example are:

$$p_i = p_{icr} = (p_{icr}/p_o)\, p_o \qquad (21)$$

$$p_i = p_{icr} = 0.2577 \times 50 = 12.89 \text{ MPa}$$

$$u_i = \bar{u}_o = (\bar{u}_o/r_i)\, r_i \qquad (22)$$

$$u_i = \bar{u}_o = (104.78 \times 10^{-4})\, 2500 = 26.2 \text{ m}$$

The support capacity values, p_s, of the various designated support systems are provided in Table 3. These support capacity values are presented in Figure A-1(c) by vertical lines positioned at the calculated corresponding supported radial deformation values, u_s.

For example, let us consider a 5 cm thick shotcrete support with a concrete compressive strength of 35 MPa after 28 days (Designation No. 2). From Table 3 the supporting capacity is p_s = 0.68 MPa. The supported radial deformation value is calculated by the following steps:

$$c = \frac{p_s b}{p_{icr}} \qquad (23)$$

$$c = \frac{0.68 \times 92.24 \times 10^{-4}}{12.89} = 4.86 \times 10^{-4}$$

and then

$$u_s = \left(\bar{u}_o/r_i - c\right) r_i \qquad (24)$$

$$u_s = (104.78 \times 10^{-4} - 4.86 \times 10^{-4})\ 2500 = 25.00 \text{ mm}$$

This calculated supported radial deformation value is applicable if the radius of opening is 2.5 m which is supported by shotcrete lining (Designation No. 2) and if the in situ stress value is 50 MPa. This in situ stress, at the given stress gradient, is equivalent to the depth of 50/0.0275 = 1818.2 m.

In order to provide the required solution to the given problem, the previously demonstrated calculations must be repeated for the various values of the following interacting variables: the in situ stress (expressed in depth), the radius of opening and the support system.

The results of the repeated calculations, obtained with the systematically changed variables, are conveniently summarized by the supported radial deformation-depth (in situ stress) curves.

Figure A-2 shows the supported radial deformation-depth curves for shotcrete linings. For each radius value in question, there are two curves. They refer the two shotcrete mixes specified in Table 3 under Designation No. 1 and 2.

The detailed calculations, demonstrated above, were for the opening with a radius of $r_i = 2.5$. Furthermore, the calculations were performed for the in situ stress value of 50 MPa. These calculations yielded the result that in case of the shotcrete lining, with the specification under Designation No. 2, the supported radial deformation of this opening is $u_s = 25.0$ mm, at the depth of $d = 1818.2$ m.

While every other conditions kept unchanged, a series of repeated calculations with various in situ stress values resulted the continuous curve, referred to as the supported radial deformation-depth (u_s vs d), curve. It is shown in Figure A-2 under r_i = 2.5 m, by the curve marked by Designation No. 2. The supported radial deformation-depth curve which is marked by Designation No. 1 was resulted by another series of calculation, using the support capacity value of p_s = 0.27 MPa (see Table 3) of the shotcrete mix.

The same procedures were followed to establish the supported radial deformation-depth curves for the other requested opening sizes, i.e. r_i = 1.0 m, r_i = 5.0 m and r_i = 10.0 m. All of the u_s vs d curves, obtained for the two shotcrete mix types, are shown in Figure A-2.

Figure A-3 shows the u_s vs d curves obtained for the 30 cm and the 50 cm thick concrete linings, marked by Designation No. 3 and 4, respectively.

The supported radial deformation-depth curves of the three types of steel sets supports, as specified under Designation No. 5, 6 and 7 in Table 3, are shown in Figure A-4.

The supported radial deformation-depth curves presented in Figure A-5 are for the rock bolt support. It should be remembered that the supported radial deformation values, u_s, in the case of rock bolts is assumed to be equal to $\bar{u}_o$. With other words, the effectiveness of the rock bolt support is attributed to other mechanism than to the low support pressure values of p_s that can be accounted for any rock bolt pattern (see Table 4).

The effective and economic depths of these traditional support systems are to be established from the obtained u_s vs d curves.

From the stability analysis the critical in situ stress for the given rock mass is p_{ocr} = 15.81 MPa. This pressure is equivalent to 586 m depth. The rock mass, regardless of the size of the opening, can be

considered as self-supporting down to this depth. The starting point of an u_s vs d curve is at the depth of 586 m.

Below this depth the openings need some type of support. The limiting depth for an effective support, for each supporting system, is given by the depth where the radial deformation value of the u_s vs d curve is equal to u_{ult}.

Let us assume that the allowable radial deformation for the given rock mass is 20 mm. The vertical line in Figure A-2 represents this u_{ult} = 20 mm. It intersects the u_s vs d curves at 1160 m and 1330 m depths. These are the limiting depths for the shotcrete lining of lower and higher strength mix, respectively, for an opening with a radius of r_i = 2.5 m.

The limiting depth of support effectiveness, for the mechanically anchored rock bolt support as shown in Figure A-2, is 1200 m, providing that u_{ult} = 20 mm. However, one could argue that in case of frictional and grouted bolting the allowable radial deformation can be higher, due to the flexible nature of these types of boltings. If it is assumed that u_{ult} = 25 mm, then the limiting depth of bolting would increase to 1730 m.

For the heavier support systems the calculations could result negative supported deformation values. As shown in Figure A-3 this is the situation in case of concrete linings, for all the considered opening sizes. However, in case of steel sets supports, as shown in Figure A-4, negative values for u_s were obtained only for the smaller opening sizes (i.e. r_i = 1.0, r_i = 2.5 and r_i = 5.0 m) and only for the heaviest sets (Designation No. 7).

The negative value of u_s is an indication of excess supporting capacity. For the depths with negative values of u_s the use of the support system is uneconomic. The system may become economic for the depths below d_{min}, which is defined by the intersection point between the u_s vs d curve and the vertical axis of u_s = 0. For example, in case of

an opening with r_i = 2.5 m, as shown in Figure A-3, the economic use of the 30 cm and 50 cm thick concrete linings begins at the depth values of 1080 m and 1220 m, respectively.

The requested design chart which can be used as a guide for support system selection, in a limestone formation characterized by the input data listed at the beginning of the presented detailed example, is given by Figure A-6.

The design chart provides direct answer on the effective use of each support system, applicable for the rock conditions and opening sizes specified by the input data.

The usefulness of the chart becomes evident by discussing some of the presented information. Let us consider, for example, the steel sets support system by comparing the effective and economic use of the lightest (Designation No. 5) and of the heaviest (Designation No. 7) types of steel sets.

The given rock mass is self-supporting down to a depth of 586 m. If the radius is 1.0 m the lightest steel sets can be used as an effective support system to a depth over 3000 m. However, for an opening with 2.5 m radius, the lightest steel sets support can provide effective support only to the depth level of 1170 m. The limiting effective depth further decreases to 720 m and 640 m for the opening sizes of r_i = 5.0 m and r_i = 10.0 m, respectively.

If the radius is 1.0 m, the heaviest steel sets would provide an uneconomic support down to 1300 m, because of its excess support capacity. Below that it may become an economic support system and then can be effectively used well over 3000 m depth. For an opening with r_i = 2.5 m, the heaviest steel sets can provide and effective and economic support between the depths of 870 m and 1700 m. The effective and economic support depth range for an opening with r_i = 5.0 m is between 750 m and 950 m. If the radius of the opening is 10.0 m, the heaviest steel sets can be effectively used to the depth value of 680 m.

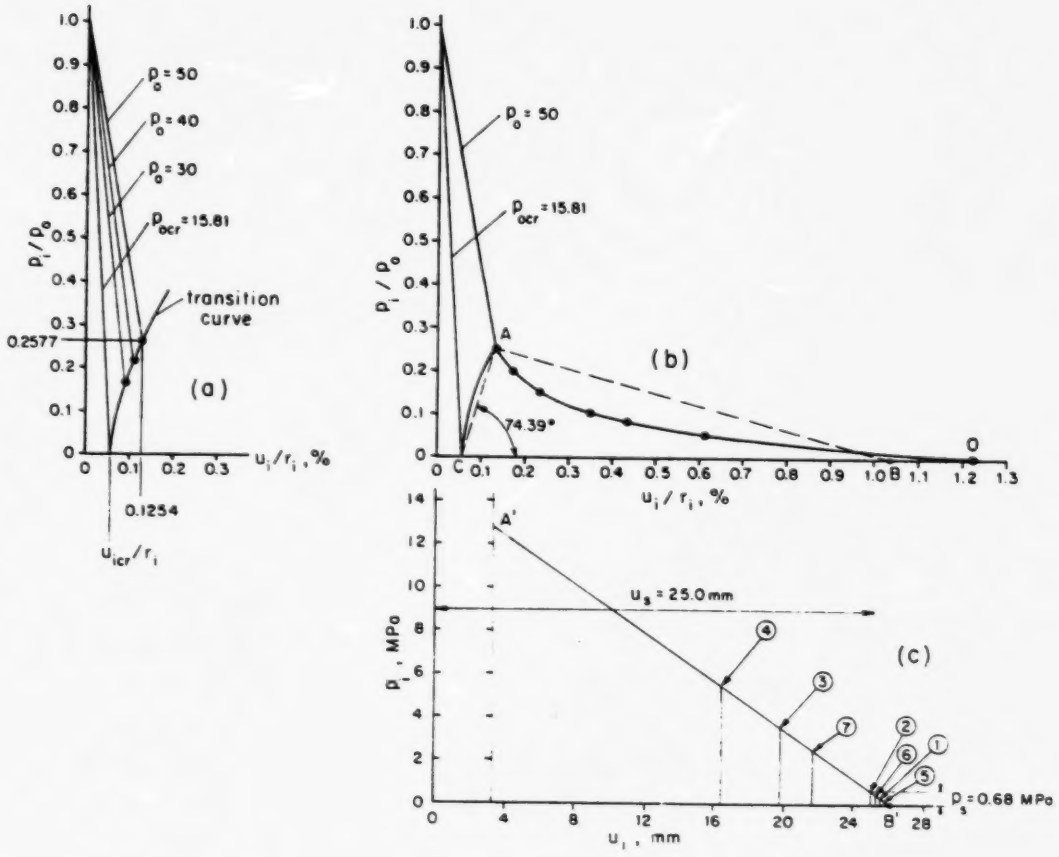


Fig. A-1 - Load-deformation reaction and support pressure - radial deformation curves.

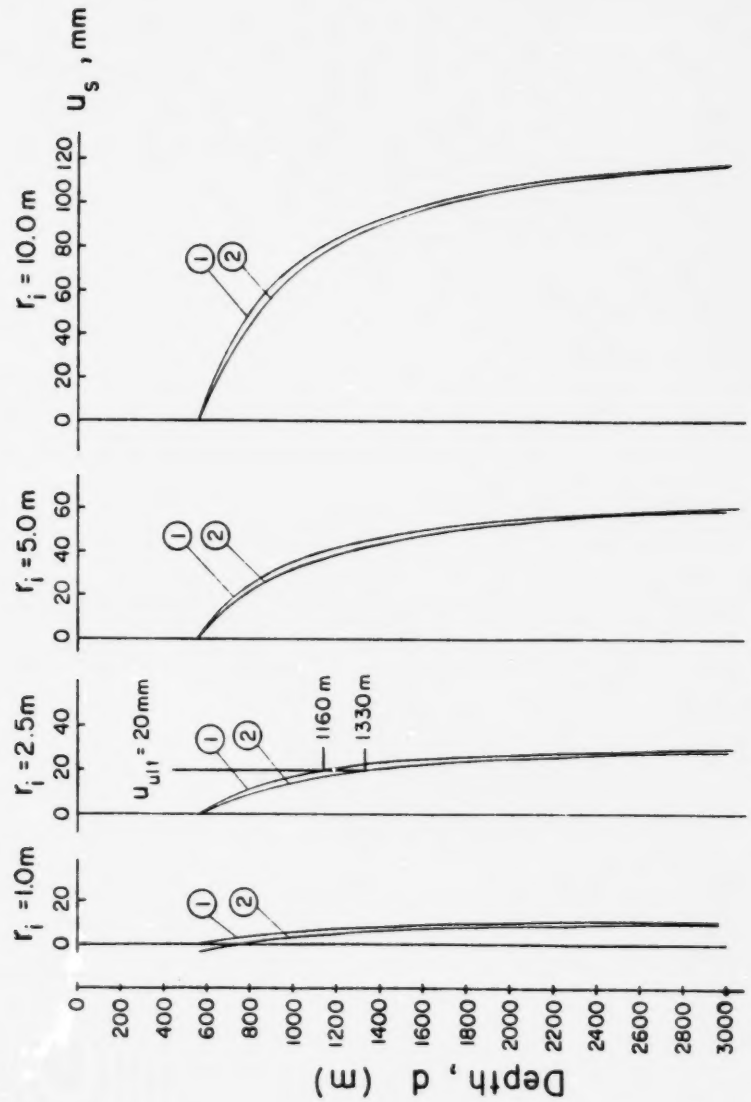


Fig. A-2 - Supported radial deformation - depth curves for shotcrete linings.

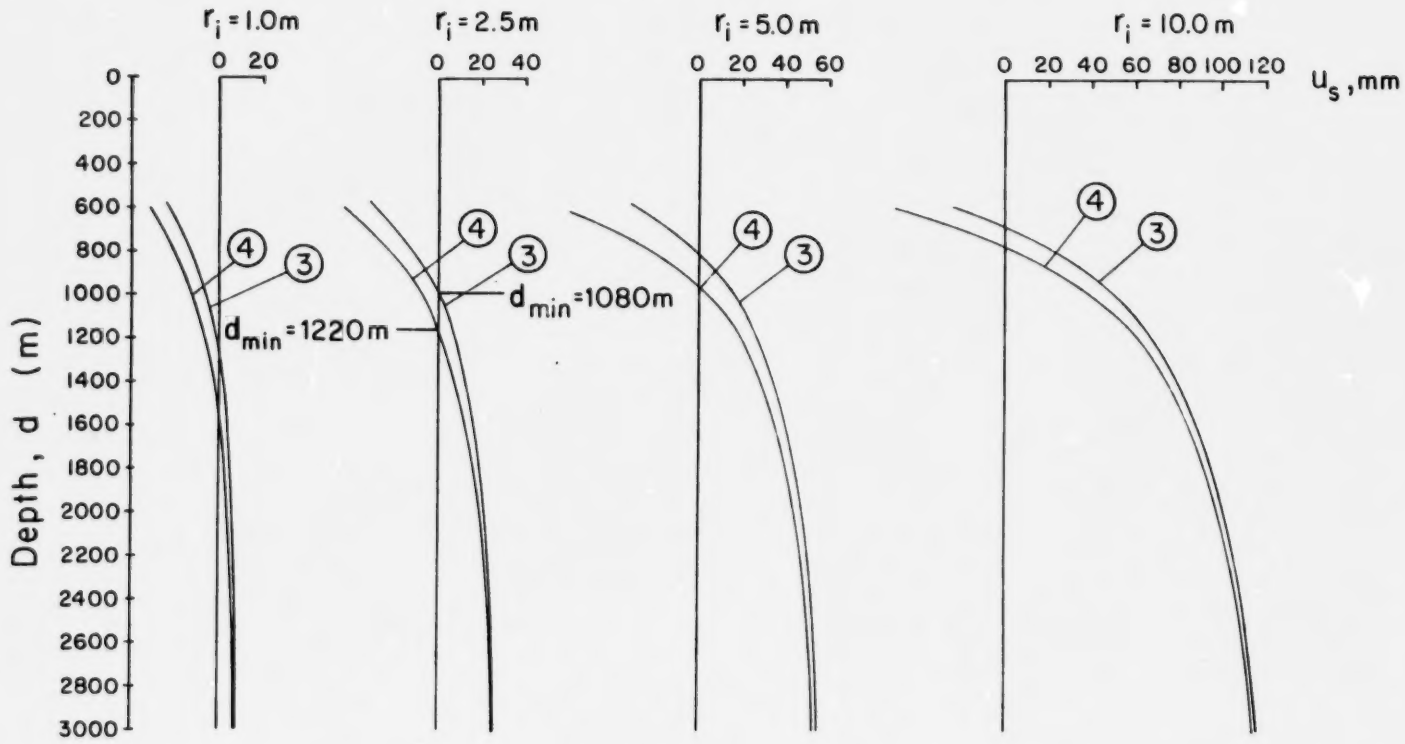


Fig. A-3 - Supported radial deformation - depth curves for concrete linings.

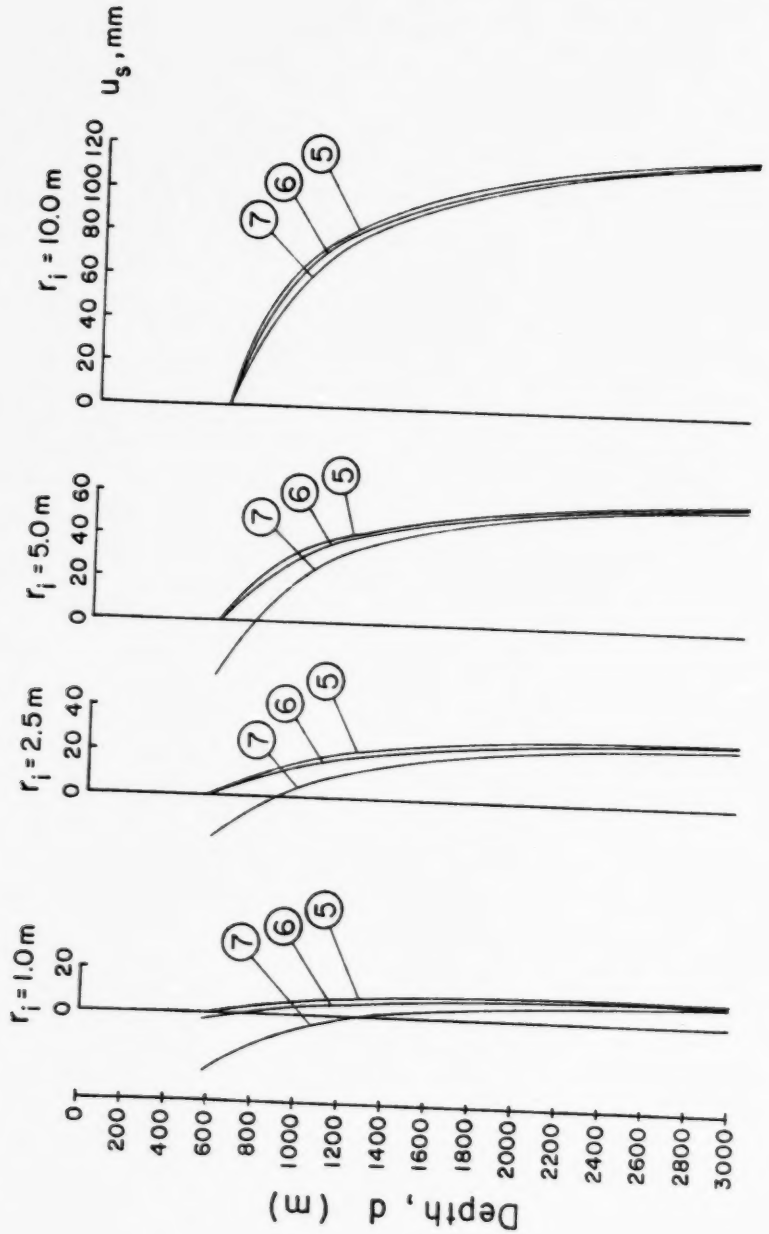


Fig. A-4 - Supported radial deformation - depth curves for steel sets supports.

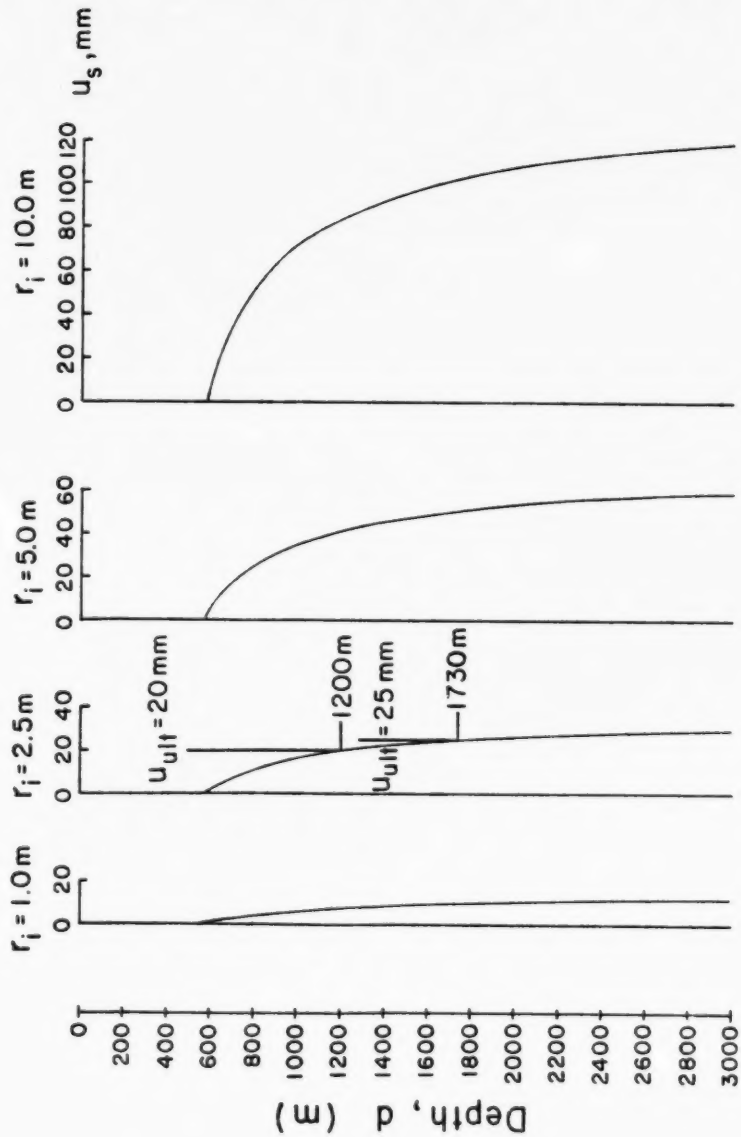


Fig. A-5 - Supported radial deformation - depth curves for rock bolts.

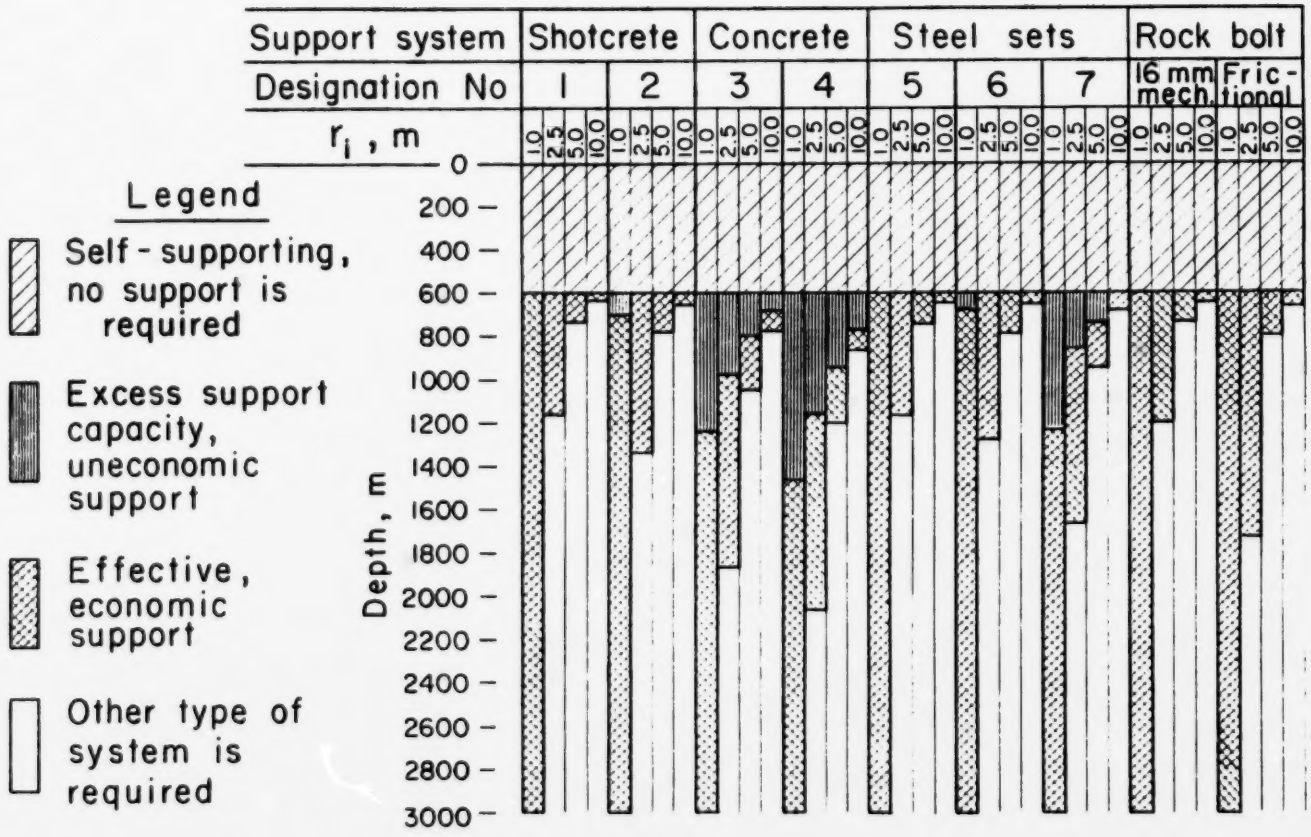


Fig. A-6 - Design charts of support systems.